observed to have horizontal cracks along the interface between the top of the wall and the floor slab above.

7.2.3 Preliminary Classification (by Observation) of Component Types, Behavior Modes, and Damage Severity

The first critical step in interpreting component damage records is to identify the components within the structural element under investigation. In this case, the example building is reinforced concrete, so the summary of relevant component types is found in Sections 2.4 and 5.2.1 of FEMA 306.

Component types, Table 5-1 of FEMA 306

7.2.3.1 Component Types

The first pass in the identification process is conducted by observation, keeping in mind that the definition of a component type is not a function of the geometry alone, but of the governing mechanism of lateral deformation for the entire element or structure. Thus the identification of structural components requires consideration of the wall element over multiple floor levels. Complete diagrams showing the crack pattern over multiple floor levels such as the ones shown in the attached damage records shown in Figures 7-6, 7-7 and Damage Records D1 through D19 (Appendix A) are essential.

For the typical coupled wall elements of the example building, shown in Figure 7-7, a survey of the element geometry and the general pattern of damage suggests that the beams over the openings may be classified as weaker coupling beams (RC3), and that the wall piers flanking the openings will behave as two-story cantilever components (RC1). The thought process that leads to this conclusion includes the recognition that the beam elements are likely to be weaker than the walls on either side of the coupling beams, as well as a mental visualization of the lateral deformation of the walls and the attendant large deformation demands on the beams. As shown in Figure 7-6, the solid reinforced concrete wall component is type RC1.

Component identification, Section 2.4 of FEMA 306

7.2.3.2 Behavior Modes and Damage Severity

Once the component types have been identified, an initial classification of the behavior modes and damage severity may be made by inspecting the visible damage with reference to the component damage classification guides. Tables 5-1, 5-2, and 5-3 of FEMA 306 are also helpful in identifying behavior modes appropriate to the identified components.

Behavior modes, Table 5-2 of FEMA 306

For the typical coupled wall shown in Figure 7-7, the coupling beam component (RC3) on the second floor is observed to have light diagonal (shear) cracking, with little or no evidence of flexural cracking. As is typical of a building designed in the late 1950s, the coupling beam does not contain diagonal reinforcement, or even sufficient stirrup reinforcement, so mode A (ductile flexure) may be safely eliminated. The diagonal cracks then suggest that the behavior mode may be either mode B (flexure/diagonal tension) or mode H (preemptive diagonal tension). At the first floor coupling beams, the damage is more severe, but the behavior mode still appears to be either B or H.

In the first floor coupling beam, identification of the damage severity is relatively straightforward: the observed damage would be classified as Heavy regardless of the behavior mode. In many cases, however, the damage severity level may depend on the behavior mode. In the second floor coupling beam, for example, the damage would be classified as Insignificant if the behavior mode is identified as B (flexure followed by diagonal tension), but as Moderate if the behavior mode is identified as H (preemptive diagonal tension).

Component Guides, Section 5.5 of FEMA 306

Similarly, the wall piers of the coupled walls (RC1) have light diagonal cracking, which may be indicative of early stages of mode B (flexure/diagonal tension), early stages of mode C (flexure/diagonal compression) or more advanced stages of mode H (preemptive diagonal tension). In the first two cases, damage would be classified as Insignificant, while in the last case, damage would be classified as Moderate.

It is often not possible to distinguish between the different behavior modes, and hence the damage severity, without some analysis. This is particularly important for lower levels of damage where different modes may look very much alike, but which have different response at higher levels of damage. Consider, for example, modes B and H. The flexural cracks that initiate mode B response may have closed and become nearly invisible. The light diagonal cracking that occurs at the outset of both modes B and H will then be indistinguishable from one another, and only analysis of the section will differentiate the two modes, and hence the severity of damage. In other cases, the differences between modes are of less impor-

Verification loop, Figure 1-3 of FEMA 306

tance. Modes B and C are physically different, but have a similar effect on the stiffness, strength, and deformation capacity of the component at all levels of damage severity.

7.2.4 Final Classification (by Analysis) of Component Type, Behavior Mode and Damage Severity

In the previous section, component type, behavior mode, and damage severity were preliminarily defined based only on observation. In this section, those definitions are verified by calculation. In practice, iterations between observation and analysis may be needed to interpret correctly the seismic response and damage.

7.2.4.1 Expected Strength

Expected strength, Section 3.6 of FEMA 306

The expected pre-earthquake strengths for each of the components were calculated using the FEMA 306 Section 3.6 procedures. The design concrete strength was shown on the drawings to be 3000 psi. According to the discussion in FEMA 306, Section 5.3.2, expected concrete strengths ranging from 1.0 to 2.3 times the specified strength are not unrealistic. In the example building, concrete strength was suspect, so tests were conducted which revealed that expected strength was, in fact, near the specified strength. For the purposes of the following analysis an expected strength of 3000 psi was assumed. Based on the drawing notes, reinforcing bars had a specified yield strength of 40 ksi. The expected strength of the reinforcing bars was assumed to be greater than the nominal yield strength by a factor of 1.25, so a value of 50 ksi was used for the yield strength in all calculations. If, during the course of the analysis, it becomes difficult to reconcile analytically determined behavior modes with observed damage, assumed values for material strength may need to be re-evaluated or verified through tests.

There are two typical element types in the lateral-force-resisting system, solid walls and coupled walls. The following sections describe the details of the calculations and methodology used to classify the components of these elements.

7.2.4.2 Example 1 – Solid Wall (2B-2C)

Once a preliminary damage classification has been made by visual observation, it will generally be necessary to perform some analysis to distinguish between behavior modes that are different but visually similar. As a first example, consider the damage record for the wall shown in Figure 7-6. The wall is 12 inches thick with 18-inch square boundary elements at each end. The wall length from center to center of the boundary elements is 26 feet, and the story height is 13 feet-6 inches. Note that the wall is L-shaped in plan and has a 26-foot return along line B.

Component Type. The definition of this wall as a single RC1 component (isolated wall or stronger wall pier) is easily and intuitively verified by sketching the inelastic deformation mechanism for the wall and its surrounding structure. The slabs framing into the wall clearly do not have the stiffness or strength to force a "weaker wall" type of behavior. The wall is therefore a single component with a height of 27 feet.

Component guides, RC1B, RC1C, and RC2H, Section 5.5 of FEMA 306

Behavior Mode. The preliminary classification identified four possible behavior modes for this component that were consistent with the component type and the observed damage: mode B (flexure/diagonal tension), mode C (flexure/diagonal compression), mode H (preemptive diagonal tension), and mode M (foundation rocking). For each of these behavior modes, Component Guides provide, in addition to the visual description of the different behavior modes, guidance in the analytical steps required to verify a particular behavior mode. See for example the Component Damage Classification Guide RC1B under "How to distinguish behavior mode by analysis". Based on the recommendations of the guide, the shear associated with the development of the maximum strength in flexure, diagonal tension, web crushing, and foundation rocking were calculated. Calculation results are summarized in Table 7-1. Selected details of the calculations are provided in the box on page 192.

The relationship between capacities of the different potential behavior modes defines the governing component behavior mode. Initially, consider the first five modes listed in Table 7-1, temporarily neglecting the overturning (foundation rocking) response. Because the wall is flanged, its response depends on the direction of seismic force, and the flexural capacity must be calculated for each direction. It is possible that a different behavior mode will govern in each of the two different loading directions. In this example, the diagonal tension strength at low ductility is less than the flexural strength

Table 7-1 Capacity of Potential Behavior Modes for Typical Solid Wall (2B-2C)

Behavior Mode	Shear Capacity *(kips)*	FEMA 306 Reference	Comments
Flexure (modes A & B) – flange in compression M_e = 31,300 k-ft	1570*	Sect. 5.3.5	All distributed reinforcement is included in the calculation of flexural strength, as is the contribution of the flange reinforcement.
Flexure (A & B) – flange in tension M_e = 44,600 k-ft	2230*	Sect. 5.3.5	
Diagonal Tension (B & H) – at low flexural ductility	1350	Sect. 5.3.6b	Low ductility implies $\mu \leq 2$ and high ductility implies $\mu \geq 5$, but for this example the exact displacement ductility is not important. Capacity at high ductility does not govern, since flexural yielding does not occur.
Diagonal Tension (B) – at high flexural ductility	851	Sect. 5.3.6b	
Web crushing (C)	2560	Sect. 5.3.6c	
Overturning (M) – flange in compression M_e = 6,860 k-ft	343	Sect. 5.2.6	When the flange is in tension, the vertical load includes dead load contribution of flange.
Overturning (M) – flange in tension M_e = 18,000 k-ft	923	Sect. 5.2.6	

* Shear associated with development of the moment strength

in either loading direction, so mode H (preemptive diagonal tension) appears to be the governing the behavior mode. In either direction, web crushing can be eliminated as a potential behavior mode since its capacity is greater than that of all of the other modes. In the absence of overturning, mode H would therefore be selected as the behavior mode for this component.

Additional calculations indicate, however, that foundation rocking (overturning of the wall and its foundation) will occur before the other failure modes can develop. This is indicated in the last two rows of Table 7-1, where overturning capacity with the flange in compression is shown to be less than other behavior modes. As shown in the example calculations (see sidebar), the foundation rocking capacity is based on the static overturning force associated with all tributary gravity loads. In reality, there are a number of factors that would increase the force required to overturn the wall, so the calculated value may be a lower bound. For example, as the foundation lifts, it will pick up an increasing tributary area of the surrounding slabs, thus increasing the restoring force. However, the overturning value calculated is sufficiently less than the other behavior modes to suggest that damage will be limited by rocking on the foundation. Mode M is therefore the behavior mode for the wall.

__Damage Severity.__ The identification of the rocking behavior mode is important, because the damage severity is different for mode M than for mode H. While there is no explicit Component Damage Classification Guide provided for the rocking mode—the component may be considered as roughly analogous to the portion of a flexural wall (mode A) above the plastic hinge region—there is a ductile fuse in the structure below the component in question that will prevent the development of the brittle, force-controlled behavior mode H by limiting the development of additional seismic force. Using this analogy, and Component Guide RC1A, the damage severity is classified as Insignificant. Without the rocking mechanism, the behavior mode would be classified as H, and the damage severity would be Moderate rather than Insignificant. It is important to note

EXAMPLE CALCULATIONS FOR DIFFERENT BEHAVIOR MODES
EXAMPLE 1 – SOLID WALL

Flexure:

The boundary elements at each end of the wall have 4-#10 and 7-#11 bars. The vertical wall reinforcment is #4 bars at 13" on center in each face. An approximation of the flexural capacity with the flange in compression may be made, assuming that all the steel in the tension boundary and all the wall vertical steel is yielding, as follows:

$$\begin{aligned} & \qquad\quad \underline{\text{Boundary}} \qquad\quad \underline{\text{Wall Verts.}} \qquad\quad \underline{\text{Dead Load}} \\ M_{e(comp.)} &= A_s f_{ye}\, l_{wall} + A_{sv} f_{ye}\, l_{wall}/2 + P_{DL\,(wall)}\, l_{wall}/2 \\ &= (15.3)\,50\,(26) + (9.2)\,50\,(13) + (419)\,13 \\ &= 31{,}300 \text{ k-ft} \end{aligned}$$

With the flange in tension the capacity increases because of the yielding of the wall vertical reinforcing in the effective flange width assumed to be one half the effective wall height (*M/V*) plus the wall thickness, or about ten feet. The capacity also increases because of the additional dead load resistance of the flange. An approximation of the flexural capacity with the flange in tension is then:

$$\begin{aligned} & \qquad\qquad\qquad\quad \underline{\text{Flange Verts.}} \qquad \underline{\text{Flange Dead Load}} \\ M_{e(ten.)} &= M_{e(comp.)} + A_{sv} f_{ye}\, l_{wall} + P_{DL(flange)}\, l_{wall} \\ &= 31{,}317 + (3.8)\,50\,(26) + (320)\,26 \\ &= 44{,}600 \text{ k-ft} \end{aligned}$$

These approximations for moment capacities were checked using strain compatibility calculations and found to be acceptable. Using an *M/V* ratio of 20 ft the shear forces associated with the moment capacities are 1570 k (flange in compression) and 2230 k (flange in tension).

Diagonal Tension (Shear Strength):

In order to include the effect of axial load on shear strength, and the potential degradation of the shear in plastic hinge zones, the equations recommended in Section 5.3.6b of FEMA 306 were used to calculate the diagonal tension strength.

$$V_n = V_c + V_s + V_p$$

An M/V ratio of 20 feet was used (approximately 0.75 times the component height) based on the analysis results for shear and moment.

$A_s = 41.2 \text{ in}^2$
$A_g = 4176 \text{ in}^2$
$\rho_s = 0.0098$

Thus Equations 5-3 and 5-4 of FEMA 306 yield

$\alpha = 1.5$ $\qquad k_{rc} = 3.5$ (low ductility)
$\beta = 0.7$ $\qquad k_{rc} = 0.6$ (high ductility)

and the concrete contribution (Equation 5-2) becomes

$$V_c = \alpha\beta k_{rc}\sqrt{f'_{ce}}\, b_w (0.8 l_w)$$

$V_c = 605$ kips at low ductility demand
$V_c = 104$ kips at high ductility demand

The steel contribution is given by Equation 5-5:

$$V_s = \rho_n f_{ye} b_w h_d$$

where $\rho_n = .00256$, $f_{ye} = 50$ ksi, $b_w = 12"$, and h_d is limited by the component height of 27'-0". Thus

$V_s = 498$ kips

The axial load contribution is given by Equation 5-6. Considering only the structure dead load tributary to the wall (419 kips) V_p becomes

$$V_p = \frac{(l_w - c)N_u}{(2M/V)} = 264 \text{ kips (flange in compression)}$$

$$= 249 \text{ kips (flange in tension)}$$

NOTE: $c = 16.8$ in. (flange in compression), $c = 33$ in. (flange in tension)

Therefore, Equation 5-1 for the diagonal tension strength gives a value of 1352 kips at low ductility demand, and 851 kips at high ductility demand, both with the flange in tension.

Diagonal Compression (Web Crushing):

The web crushing strength is given by Equation 5-7. This equation requires an estimate of the drift ratio to which the component is subjected, with increasing drift corresponding to a decrease in capacity. An upper bound estimate of 1 percent drift is assumed, to get a lower bound on the web crushing strength:

$$V_{wc} = \frac{1.8 f'_{ce} b_w (0.8 l_w)}{1 + \left(600 - 2000\dfrac{N_u}{A_g f'_{ce}}\right)\delta} = 2560 \text{ kips}$$

Foundation Rocking (Overturning):

The static overturning calculation includes not only the dead weight of the wall and tributary slabs at the 2nd floor and roof, but also a tributary area of the slab on grade (496 kips total) and the foundation weight (16 kips per footing). When the wall flange is in tension, the weight of the flange and additional DL are included.

$$V_{ot} = \frac{1}{(M/V)} M_{ot} = \frac{1}{20}(496k(26'/2) + 16k(26'))$$

$= 343$ kips (flange in compression)

$$V_{ot} = \frac{1}{(M/V)} M_{ot}$$

$$= \frac{1}{20}(496k(26'/2) + 446k(26') + 16k(26'))$$

$= 923$ kips (flange in tension)

that the damage severity is not a function of the observed crack pattern alone – the governing behavior mode must be known before a judgement of the damage severity can be made.

7.2.4.3 Example 2 – Coupled Wall (7L-7M)

As an example of the second typical wall element type, consider the damage record for the coupled wall shown in Figure 7-7. Like the solid wall example, the wall is 12 inches wide with 18-inch-square boundary elements at each end. However, there is a 6'-6" wide by 7'-3" tall opening in the center of the wall at each floor. The wall length from center to center of the boundary elements is 26 feet, and the story height is 13'-6". The coupled wall has an L-shaped plan with a 26-foot flange along line M. The coupling beam and wall are similar to the example shown in Figure 7-5, except that this particular coupled wall is discontinuous below the first floor and is supported on 24-inch-square reinforced-concrete columns at the basement.

__Component Type.__ Visual observation leads to the division of this structural element into two RC1 wall piers and two RC3 coupling beams. Analysis will verify that the beams are weaker than the walls, and thus that the initial classification is valid.

__Behavior Mode.__ In the preliminary classification, the coupling beams were designated by observation as mode B (flexure / diagonal tension) or mode H (preemptive diagonal tension), and the wall piers were designated as mode B (flexure / diagonal tension), mode C (flexure / diagonal compression), mode H (preemptive diagonal tension), or mode N (individual pier rocking). As in the first example, the shears associated with the development of the maximum strength in flexure, diagonal tension, and web crushing were calculated, with results summarized in Tables 7-2 and 7-3. Selected details of the calculations are provided for reference on pages 196 through 198.

Looking first at the RC3 coupling beam component, the calculation results shown in Table 7-2 indicate that the shear strength will be reached before the development of the moment strength, even at low ductility levels, so the behavior mode H (preemptive diagonal tension) governs.

For the RC1 wall pier components, the calculations and discussions that follow show that behavior mode N, individual pier rocking, governs the seismic response. For the piers of the coupled wall, which discontinue below the first floor and are supported on basement columns, this behavior mode involves the yielding in flexure of the basement columns and the coupling beams reaching their capacity in shear. The wall pier rotates about the supporting column in a manner similar to

Table 7-2 Capacity of Potential Behavior Modes for Typical Coupling Beam

Coupling Beams RC3 Behavior Mode	Limiting Component Shear (kips)	FEMA 306 Reference	Comments
Flexure (mode A) M_e = 1210 k-ft	373*	Sect. 5.3.5	Note that slab reinforcement was ignored in the calculation of the beam flexure capacity. Since preemptive shear governs (242 < 373), this is irrelevant. A more accurate calculation would be warranted if the capacities in the different modes were similar.
Diagonal Tension (B and H) – at low flexural ductility	242	Sect. 5.3.6b	Governing capacity
Diagonal Tension (B) – at high flexural ductility	137	Sect. 5.3.6b	This capacity does not govern since flexural yielding does not occur.
Sliding Shear (D)	150	Sect. 5.3.6c	This mode is unlikely since it typically occurs after flexural yielding. Such yielding is not expected since preemptive diagonal tension governs over flexural response.

* Component shear in beam associated with development of the component moment strength

Table 7-3 Shear Capacities for Potential Behavior Modes of Wall Pier (RC1) Components in Coupled Wall

Potential Behavior Mode	Limiting Component Shear *(kips)*	FEMA 306 Reference	Notes
Flexure(mode A)	See notes*	Sect. 5.3.5	*In example calculations, moment capacities are compared to moment demands corresponding to mode N. Flexure is shown not to govern.
Diagonal Tension (mode B and H) at Low Flexural Ductility		Sect. 5.3.6b	Limiting shears are compared to those for behavior mode N. To consider redistribution of lateral forces, the sum of shears for the two wall piers is considered.
RC1@7L-load to east	690		
RC1@7L-load to west	311		
RC1@7M-load to east	328		
RC1@7M-load to west	692		
Diagonal Tension (mode B) at High Flexural Ductility		Sect. 5.3.6b	These capacities do not govern, since flexural yielding does not occur.
RC1@7L-load to east	470		
RC1@7L-load to west	163		
RC1@7M-load to east	166		
RC1@7M-load to west	472		
Web Crushing (mode C)		Sect. 5.3.6c	Web crushing not applicable for low axial load or tension.
RC1@7L-load to east	1710		
RC1@7M-load to west	1810		
Rotation about Column (mode N)			Shear in piers is limited by capacity of coupling beam (RC3) components.
RC1@7L-load to east	330		
RC1@7L-load to west	300		
RC1@7M-load to east	300		
RC1@7M-load to west	330		

foundation rocking. Free body diagrams corresponding to this mechanism and behavior mode are shown in the example calculations that follow.

Comparison of the moment demands corresponding to the behavior mode N to moment capacities of the wall pier sections is shown in the example calculations. The moment demands are well below the moment capacities, indicating that flexural yielding will not occur. This eliminates modes B (flexure/diagonal tension) and C (flexure/diagonal compression) as possible behavior modes.

The limiting component shears associated with possible behavior modes for the wall piers are summarized in Table 7-3. The table verifies that the web crushing (diagonal compression) can be eliminated as a possible behavior mode because the capacity is much higher than that corresponding to other behavior modes. Behavior mode H, preemptive diagonal tension, is investigated by comparing the limiting shears to those of mode N.

Diagonal tension capacities at high ductility are only relevant for the combined flexure/diagonal tension behavior mode, which will not occur since flexural yielding and the consequent degradation of the V_c component of shear strength does not occur. The relevant diagonal tension capacities are those at low ductility.

The diagonal tension capacities of 311k (RC1@7L-load to west) to 328k (RC1@7M-load to east) for the wall piers subject to axial tension are similar to the shear demands in the pier rotation mode after failure of the coupling beams; however, there is significant capacity of 690k (RC1@7L-load to east) to 692k (RC1@7M-load to west) in diagonal tension on the corresponding compression sides of the wall. A diagonal tension failure cannot fully develop on one side of the coupled wall without transferring lateral forces to the other side of the wall. Considering that shear can be transferred as axial forces in the coupling beam and slab according to the stiffness and strength of each wall pier, the sum of wall pier component strengths on each side of the coupled wall can be used to determine the governing behavior mode. For the individual pier rotation behavior, the associated total shear demand is 630k on the coupled wall element. For a diagonal tension behavior mode occurring in both wall piers, the associated shear capacity is 1003k to 1018k. Diagonal tension failure will not govern, since the pier rotation behavior mode occurs at a lower total lateral load. Thus, the results of the analytical calculations indicate the pier rotation (N) is the governing behavior mode for the RC1 components. This analytical conclusion agrees with field observation. The degree of diagonal cracking observed in the wall pier RC1 components is consistent with substantial shear stress, but less than that which might be expected for diagonal tension failure.

Damage Severity. For the RC3 components behaving in mode H, the damage classification guides indicate that the observed damage is Moderate in the second story and Heavy in the first story coupling beam. In the wall piers, the protection of the element by a ductile mode (similar to mode N, Foundation Rocking) in surrounding components places them in an Insignificant damage category.

7.2.5 Other Damage Observations

Several of the walls were observed to have horizontal cracks just below the roof slab and/or the second-floor slab. In addition to new cracks of this type, a few walls had pre-existing horizontal cracks below the slabs, which had been repaired by epoxy injection. The widest of these horizontal cracks occurred under the roof slab of the wall on column lines 7C-7D, as shown in the Component Damage Record D3. The engineer in the field indicated that joint movement occurred at this crack and suspected that sliding shear behavior may have occurred.

Subsequent thinking by the evaluating engineers about this observation, however, weighed against the conclusion of sliding shear behavior. The crack was not observed to extend into the boundary columns of the wall, and there was no evidence of lateral offset at the boundary columns. While the crack is located near a likely construction joint where poor construction practice can exacerbate sliding shear behavior, the crack is not located in the maximum moment region of the wall. As is indicated in FEMA 306, sliding shear behavior is most likely to occur after flexural yielding has occurred. For this wall, flexural yielding would initiate at the base of the wall where moments are at a maximum, not at the top. In any case, foundation rocking preempts flexural yielding for the typical solid wall, as indicated previously in this example. A quick calculation of sliding shear strength shows that the behavior mode is not expected to govern the wall's response.

Given this information, the damage observations are reconsidered, and it is judged that sliding movements did not occur at the horizontal crack. Therefore, the most likely explanation is that these horizontal cracks are caused by earthquake displacements in the *out-of-plane* direction of the wall. It is judged that the horizontal cracks, whose widths are less than 0.03 inches, do not significantly affect seismic response.

7.2.6 Summary of Component Classifications

7.2.6.1 Solid Walls

All wall components of the building are evaluated in a similar manner, as described in the preceding sections. In total, the building has six coupled walls plus five solid walls acting in the North-South direction, and two coupled walls plus six solid walls acting in the East-West direction. The damage records for these walls can be found in Component Damage Records D1–D19 (Appendix A).

Each solid wall is a single structural component (RC1), while each coupled wall has four components: two coupling beams (RC3) and two wall piers (RC1). Thus there are a total of 43 structural wall components in the building, as indicated in Tables 7-4 and 7-5. For each of these, the component type, behavior mode and damage severity is established as described below and shown in the tables.

EXAMPLE CALCULATIONS FOR DIFFERENT BEHAVIOR MODES
EXAMPLE 2 – COUPLED WALL (7L-7M)

COUPLING BEAMS

RC3 Flexure:

The moment strength of the coupling beams is calculated as discussed in FEMA 306, Section 5.3.5 using expected values for material properties (f'_{ce} = 3000 psi, f_{ye} = 50 ksi). The beams are 6'-3" deep, with 3 - #9 bars at top and bottom and #4 bars @ 13" on center at each face. The calculated moment capacity is 1210 k-ft. This capacity is determined using strain compatibility calculations that demonstrate that all longitudinal bars yield. The M/V ratio for the coupling beam is 3'-3", so the shear associated with development of the moment capacity at each end of the beam is 373 kips. Note that slab reinforcement is ignored in the calculation of the beam flexure capacity. It will be shown below that preemptive shear clearly governs, so this is irrelevant. However, a more accurate calculation would be warranted if the capacities in the different modes were similar.

RC3 Diagonal Tension (Shear Strength):

The equations for diagonal tension strength in Section 5.3.6b of FEMA 306 may be used for coupling beams. For beams, the axial load is not significant, thus $V_p = 0$ and Equation 5-1 becomes:

$$V_n = V_c + V_s$$

Using an M/V ratio of 3'-3" (half the clear span of the coupling beams) Equations 5-3 and 5-4 of FEMA 306 yield

$\alpha = 1.5$ $\rho_g = 0.0059$ $\kappa_{rc} = 3.5, 0.6$

$\beta = 0.61$ $\sqrt{f'_{ce}} = 55$ psi

and the concrete contribution Equation 5-2 becomes

$$V_c = \alpha\beta k_{rc}\sqrt{f'_{ce}}\,b_w(0.8l_w)$$

V_c = 127 kips at low ductility

V_c = 22 kips at high ductility

The steel contribution is given by Equation 5-5

$$V_s = \rho_n f_{ye} b_w h_d$$

where ρ_n = .00256 is based on the vertical (stirrup) reinforcement, f_{ye} = 50 ksi is the expected steel yield strength, b_w = 12", and h_d = 75" is the horizontal length over which vertical stirrup reinforcement contributes to shear strength, in this case the length of the coupling beam. Thus

V_s = 115 kips

The total diagonal tension strength is then 242 kips at low ductility, and 137 kips at high ductility.

RC3 Sliding (Sliding Shear):

FEMA 306 Section 5.3.6d gives the sliding shear strength for coupling beams at moderate ductility levels as

$$V_{sliding} = 3\left(\frac{l_n}{h}\right)\sqrt{f'_{ce}}\,b_w d = 150 \text{ kips}$$

This failure mode is generally associated with beams that are well reinforced for diagonal tension, and that undergo multiple cycles at a moderate ductility level. Since the preemptive shear failure mode governs, the sliding shear mode is not a potential failure mode.

WALL PIERS

RC1 Flexure:

The figures below show the free body diagrams of the wall for lateral forces toward the east and toward the west. In both cases it is assumed that the coupling beams and first floor slab have reached their capacities. It is also assumed that the columns beneath the first floor are yielding in flexure. These assumptions define a potential inelastic lateral mechanism for the wall. If the assumed lateral mechanism for the coupled wall is correct, the flexural capacity of the RC1 components must be sufficient to generate the diagonal tension failure in the RC3 coupling beams. The moment demand diagrams for the RC1 pier components are also shown below.

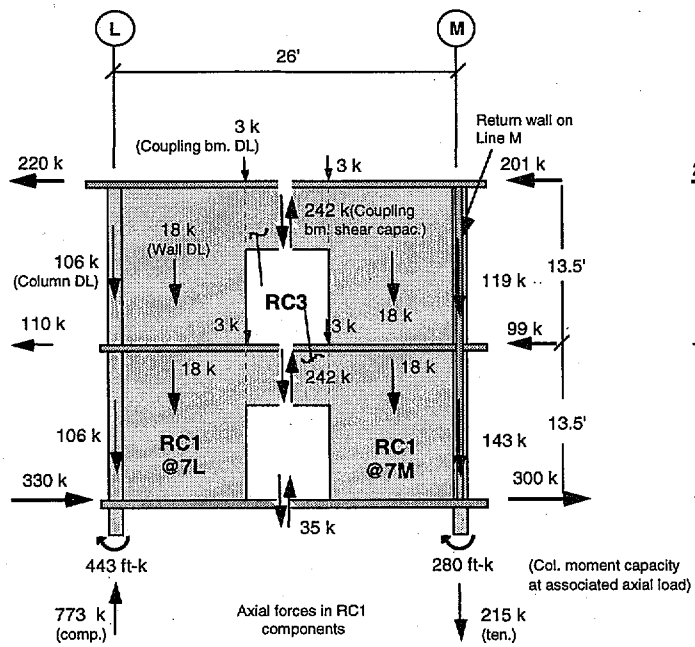
The boundary elements in the wall piers at lines L and M each contain 8-#11 vertical bars. The vertical wall reinforcing comprises #4 bars at 13" on center in each face. Using strain compatibility calculations, the moment capacities at the top and bottom of the piers (between the first floor and the top of the door opening) corresponding to the appropriate axial loads are calculated.

The moment capacity and demand for the RC1 components must be determined with respect to the same axis. For RC1@L the elastic centroid is selected. For RC1@M the elastic centroid of the component neglecting the return wall is used as the axis. When the return wall is in compression it contributes little to the flexural strength of the wall pier. However, when in tension, the reinforcment in the return increases moment strength. Therefore, in the capacity calculations, the vertical reinforcment in approximately 10 ft. of return is included. This distance is estimated in accordance with FEMA 306 Section 5.3.5b as 50% to 100% of the M/Vfor the entire wall.

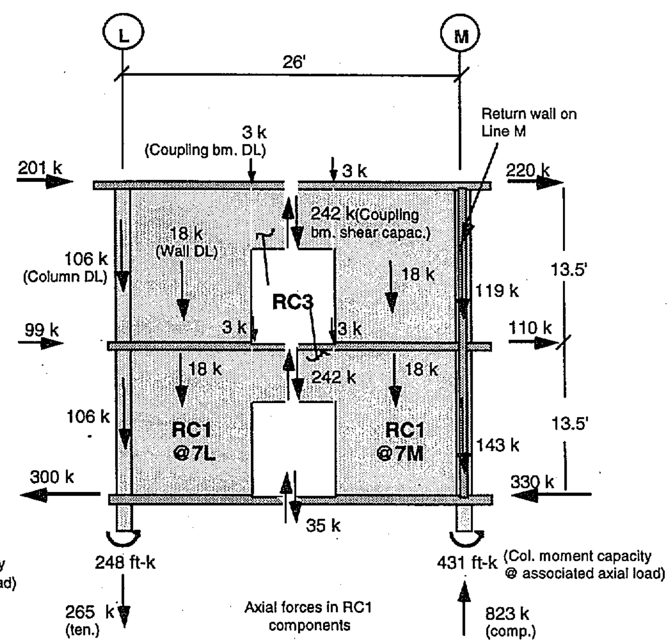
The flexural demand and capacity of the RC1 components are summarized in the following table:

CALCULATIONS FOR EXAMPLE 2 – COUPLED WALL (continued)

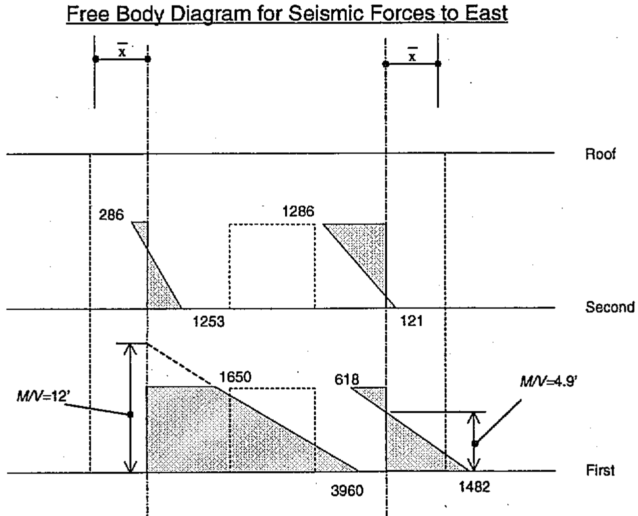
Component	Load Direction	Location on Pier	Axial Load (k,comp.+)	Moment Capacity (k-ft)	Moment Demand (k-ft)
RC1@L	East	Top	773	6470	1650
		Bottom	773	6470	3960
	West	Top	-265	2190	428
		Bottom	-265	2190	1660
RC1@M	East	Top	-215	2400	618
		Bottom	-215	7120	1480
	West	Top	823	6660	1850
		Bottom	823	6660	4160



Free Body Diagram for Seismic Forces to East

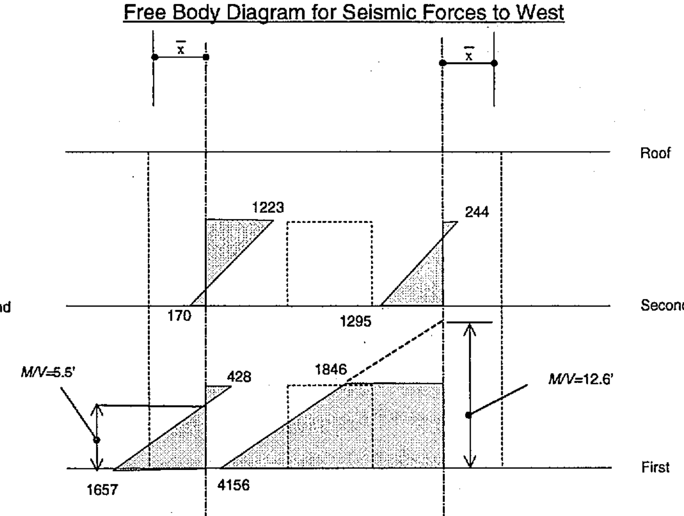


Free Body Diagram for Seismic Forces to West



Moment Diagram(@ centroid of piers) for Load to East

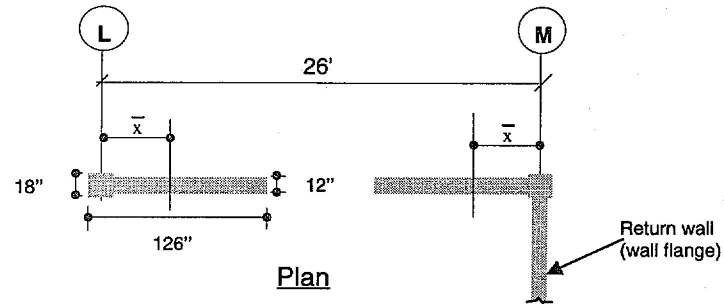
(plotted on tension side in k-ft)



Moment Diagram(@ centroid of piers) for Load to West

(plotted on tension side in k-ft)

CALCULATIONS FOR EXAMPLE 2 – COUPLED WALL (continued)



Distance to the elastic centroid from gridline:

X={ [126(12)126/2+2(18)3(9)] / [126(12)+2(18)3] } - 9

= 50.2" or 4.2'

Comp.	Load Direct.	Axial Load (k)	V_c	Reduce for Ten.	Net V_c (k)	V_s (k)	V_p (k)	Tot. V (k)	Duct.
RC1@L	East	773 (comp.)	265 45	1.0	265 45	133	292	690 470	low high
	West	-265 (ten.)	265 45	0.67	178 30	133	0	311 163	low high
RC1@M	East	-215 (ten.)	265 45	0.74	195 33	133	0	328 166	low high
	West	823 (comp.)	265 45	1.0	265 45	133	294	692 472	low high

RC1 Diagonal Tension (Shear Strength):

The equations in Section 5.3.6 of FEMA 306 were again used to calculate the diagonal tension strength.

$$V_n = V_c + V_s + V_p$$

Using the component M/V values from the moment diagrams, Equations 5-3 and 5-4 yield

$\alpha = 1.5$ $\beta = 0.76$ $\rho_g = 0.0013$

and the concrete contribution from Equation 5-2 becomes

$$V_c = \alpha\beta k_{rc}\sqrt{f'_{ce}}\,b_w(0.8l_w)$$

V_c = 265 kips at low ductility

V_c = 45 kips at high ductility

When the component experiences net axial tension ACI 318-95, eqn. 11-8 specifies the the concrete contribution to shear strength, V_c, be reduced by the factor 1-[N_u / (500 A_g)].

The steel contribution is given by Equation 5-5

$$V_s = \rho_n f_{ye} b_w h_d$$

where ρ_n = .00256, f_{ye} = 50 ksi, b_w = 12", and h_d is limited by the height of the door 7'-3". Thus

V_s = 133 kips

The compressive axial load contribution is given by Equation 5-6.

$$V_p = \frac{(l_w - c)N_u}{\left(2\dfrac{M}{V}\right)}$$

Considering all of the above contributions the diagonal tension strengths of the RC1 components are summarized in the table above:

RC1 Diagonal Compression (Web Crushing):

The web crushing strength is given by Equation 5-7. This equation requires an estimate of the drift ratio to which the component is subjected, with increasing drift decreasing the capacity. An upper bound estimate of 1% is assumed to get a lower bound on the web crushing strength:

$$V_{wc} = \frac{1.8 f'_{ce} b_w (0.8l_w)}{1 + \left(600 - 2000\dfrac{N_u}{A_g f'_{ce}}\right)\delta} = 1710 \text{ kips for RC1@7L load to East}$$

$$= 1807 \text{ kips for RC1@7M load to West}$$

Web crushing is not typically an issue for low axial loads or net tension.

The typical solid walls were calculated to behave in a foundation rocking (or overturning) mode (type M). There are no damage guides for this behavior mode. However, component behavior description in FEMA 306 considers this mode to have moderate to high ductility. The damage associated with this behavior mode may not be apparent based on the observations of the walls. Damage to other structural and nonstructural elements, such as damage to the floor slab at the base or to the beams framing into the ends of the walls, should be used to assess the severity of the mode. Since there was no significant damage to the adjacent structural and nonstructural elements, the damage severity is judged to be Insignificant.

7.2.6.2 Coupling Beams

Based on calculations, the behavior mode of the coupling beams is Preemptive Diagonal Tension (Type H). Based on the damage observations and the component guides, the damage for the coupling beams with spalling, bar-buckling, and/or significant cracking was classified as Heavy. For the coupling beams with shear cracking, but no bar-buckling or significant spalling, the damage is Moderate.

7.2.6.3 Wall Piers

The walls adjacent to the coupling beams are expected to behave in a mode of indiviudal pier rocking (type N). Thre are no Component Guides for this behavior mode. However, the component behavior description for this mode of behavior considers this mode to have moderate to high ductility. Similar to the solid shear walls, the lack of damage to the adjacent structural and nonstructural elements was used to classify the damage as Insignificant.

7.3 Evaluation by the Direct Method

Hypothetical repairs for direct method, Section 4.6 of FEMA 306

The effects of damage are quantified by the costs associated with potential repairs (component restoration measures), which if implemented, would restore the components to their pre-event condition. In the direct method, restoration measures are considered on a component-by-component basis without an analysis of global performance. It is intended to be a simple and approximate approach. The Component Damage Classification Guides in FEMA 306 are used to determine the appropriate potential repairs to restore each component.

The potential repairs required to restore the structural performance and nonstructural functionality of the building include both structural and nonstructural (e.g., cosmetic) measures for each damaged component.

7.3.1 Structural Restoration Measures

7.3.1.1 Coupling Beams

Damage guide for RC3H, Table 5-2 of FEMA 306

As shown in Tables 7-4 and 7-5, three of the coupling beams were classified as component type RC3, behavior mode H, having Heavy damage. As recommended for this component type, behavior mode, and damage severity, the component restoration measure chosen is to replace these components. The proposed repair would be to remove the concrete at the coupling beam and a portion of the floor slab, install new reinforcing bars, and cast new concrete for the wall. The new reinforcing steel in the coupling beams would be detailed in accordance with the current provisions of the governing building code for coupled shear walls, as shown in Figure 7-8.

The coupling beams with Moderate damage could be repaired by epoxy injection of all diagonal shear cracks greater than 10 mils wide, since epoxy injection is recommended for structural restoration using the damage guide for RC3H. Although it is possible to inject smaller cracks, the additional cost does not justify the marginal benefit. Since cracks as large as 12 mils can be tolerated in normal concrete structures (ACI, 1994), the unrepaired cracks should not be detrimental. The length of the cracks to be injected is estimated as 100 feet.

7.3.1.2 Solid Walls

The remaining wall components are type N or M. There are no Component Guides for these modes to indicate the appropriate repairs directly. As discussed earlier, these modes have moderate to high ductility capacity. Conservatively, the damage guide for Type B, flexure / diagonal tension, is used since this is a moderate ductility mode, analogous to the actual behavior mode. The Component Guides for the type RC1B components indicate that if cracks are less than 1/16 inch, the damage can be classified as Insignificant, and therefore structural repairs are not necessary. Two of the shear wall components had cracks that exceeded 1/16 inch. This amount of cracking would be classified as Moder-

Table 7-4 Summary of Component Type, Behavior Mode, and Damage Severity for Wall Components (North-South Direction)

Column Line	Floor	Wall Type	Component Type and Behavior Mode	Damage Severity
B / 2-3	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Moderate
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Moderate
B / 5-7	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Moderate
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Moderate
B / 10-12	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Moderate
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Moderate
B / 14-15	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Heavy
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Moderate
E / 2-3	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Moderate
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Insignificant
E / 14-15	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Moderate
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3H	Moderate
G / 7-8	First-Second	Solid	RC1M	Insignificant
G / 9-10	First-Second	Solid	RC1M	Insignificant
M / 7-8	First-Second	Solid	RC1M	Insignificant
M / 9-10	First-Second	Solid	RC1M	Insignificant
M / 8-9	Ground	Solid	RC1B	Insignificant

Table 7-5 Summary of Component Type, Behavior Mode, and Damage Severity for Wall Components (East-West Direction)

Column Line	Floor	Wall Type	Component Type and Behavior Mode	Damage Severity
7 / L-M	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Heavy
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3N	Moderate
10 / L-M	First-Second	Coupled	RC1N	Insignificant
	First	Coupled	RC3H	Heavy
	First-Second	Coupled	RC1N	Insignificant
	Second	Coupled	RC3N	Moderate
2 / B-C	First-Second	Solid	RC1M	Insignificant
2 / D-E	First-Second	Solid	RC1M	Insignificant
7 / C-D	First-Second	Solid	RC1M	Insignificant
10 / C-D	First-Second	Solid	RC1M	Insignificant
15 / B-C	First-Second	Solid	RC1M	Insignificant
15 / D-E	First-Second	Solid	RC1M	Insignificant

ate for type B behavior. Epoxy injection is recommended in the Component Damage Classification Guides for these cracks. Thus, for performance restoration by the direct method, these walls would have all of the cracks exceeding 1/16 inch repaired by injection with epoxy. The total length of crack to be injected is estimated at 22 feet.

Spalls (other than at the coupling beams that are being replaced) could be repaired by application of a concrete repair mortar to restore the visual appearance. The total volume of concrete spalls is estimated to be 3 cubic feet.

7.3.2 Nonstructural Restoration Measures

The wall components with visible cracks could be repaired by patching the cracks with plaster, and then painting the entire wall. This repair is only intended to restore the visual appearance of the wall. Restoration of other nonstructural characteristics, such as water tightness and fire protection, are not necessary in this instance.

In addition, many of the suspended ceiling tiles became dislodged and fell during the earthquake. The nonstructural repairs would include replacing the ceiling tiles.

7.3.3 Restoration Summary and Cost

Table 7-6 summarizes the performance restoration measures and estimated costs. Additional costs related to inspection, evaluation, design, management and indirect costs may also be involved.

7.4 Evaluation by Performance Analysis

The use of the direct method is limited to an estimate of the loss associated with the damaging earthquake. It cannot be used to evaluate actual performance. For these purposes, relative performance analysis as described in FEMA 306 is used. The basic procedure comprises a comparison of the anticipated performance of the building in future earthquakes in its pre-event, damaged, and repaired conditions. This comparison may be made for one or more performance objectives.

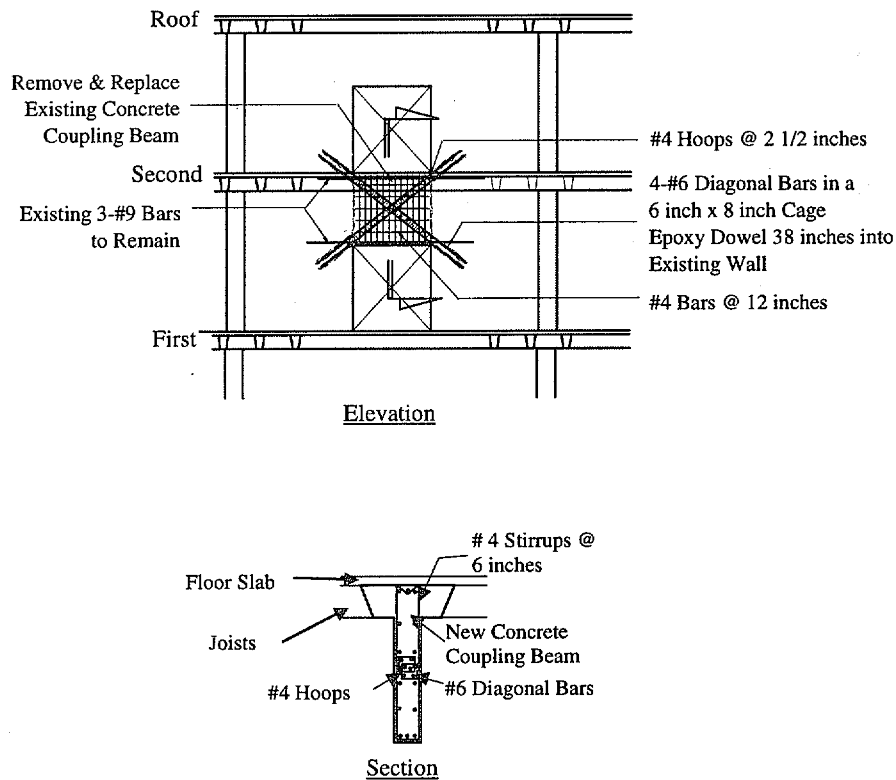


Figure 7-8 ***Detail of Coupling Beam Replacement***

Table 7-6 ***Restoration Cost Estimate by the Direct Method***

Item	Unit Cost (1997 dollars)	Quantity	Cost (1997 dollars)
Epoxy Injection	$25.00 /lin ft	122 ft	$ 3,050.
Coupling Beam Removal and Replacement	$74.00 /cu ft	122 cu ft	$ 9,028.
Patch and paint walls	$0.60 /sq ft	10,175 sq ft	$ 6,105.
Replace ceiling tiles	$2.00 /sq ft	15,000 sq ft	$30,000.
General Conditions, Fees, Overhead & Profit (@ 30%)			$14,455.
Total			$62,638.

7.4.1 Performance Objectives

Performance objectives, Section 4.2 of FEMA 306

Two performance objectives are considered in this example. The first is the *life safety* performance level, as defined in FEMA 273, for an earthquake associated with a 475-year return period (10 percent probability of exceedance in 50 years) for this site. The response spectrum for this earthquake is shown in Figure 7-9. The soil at the site was determined to be type S_C. Using the available seismic data, the spectral response at short periods (T = 0.2 sec) for this site is 1.0 g and the spectral response at 1 second is 0.56 g.

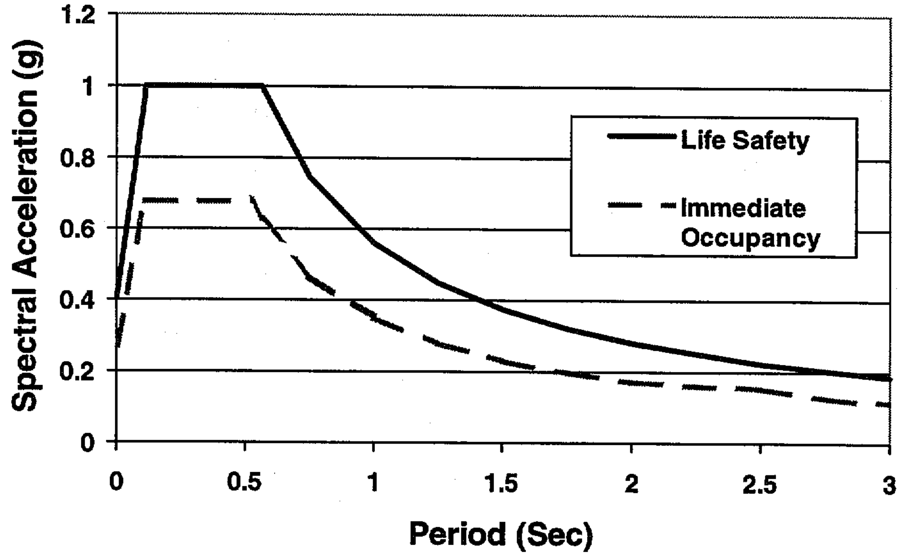


Figure 7-9 ***Response Spectra for Selected Performance Levels***

The building was also checked for *immediate occupancy* performance level using an earthquake with a 50 percent probability of exceedance in 50 years. For this earthquake, the spectral response at short periods at this site is 0.68 g and the spectral response at 1 second is 0.35 g. The response spectra for the immediate occupancy performance level is also shown in Figure 7-9.

It should be noted that these performance objectives do not necessarily correspond to the original criteria used for design of the building.

7.4.2 Nonlinear Static Analysis

7.4.2.1 Computer Model

The building is analyzed in its pre-event, post-event and repaired conditions using a three-dimensional computer model. Modeling of the building is done using the recommendations of FEMA 273 and FEMA 306. The model is subjected to a nonlinear static (pushover) analysis to assess its force/displacement response. For this example, the analysis is run only in the East-West direction, which is the direction that experienced the most significant damage.

The computer analysis program SAP2000 (CSI, 1997) is used to model the structure. The reinforced concrete walls and coupling beams are modeled using beam elements. The beam elements are located at the center of gravity of each wall section, and are given properties that represent the wall section stiffness. Rigid end offsets are used to model the joint regions in the coupled walls as shown in Figure 7-10. Small models of individual walls are used to verify that the beam elements used to model the walls have approximately the same stiffness and shear distribution as a model using shell elements for the walls. A three dimensional view of the global model is shown in Figure 7-11. The horizontal floor and roof diaphragms are modeled using beam elements, as shown in Figure 7-11, with lumped masses at the nodes.

The pushover analysis is conducted by applying static loads at the locations of the lumped masses in a vertical distribution pattern as described in the second option of Section 3.3.3.2 C, of FEMA 273. Sixty percent of the total lateral force is applied to the roof, thirty percent is applied at the second floor, and ten percent is applied at the first floor. The nodal loads are increased proportionally in progressive iterations. When elements reach their strength limit, their stiffness is iteratively reduced to an appropriate secant stiffness and the model is rerun at the same load level until no elements resist loads in excess of their calculated capacities. (Secant stiffness method, see side bar.)

The pushover analysis is continued to cover the displacement range of interest, which is based on a preliminary estimate of the maximum displacement demand. A global pushover curve is then produced.

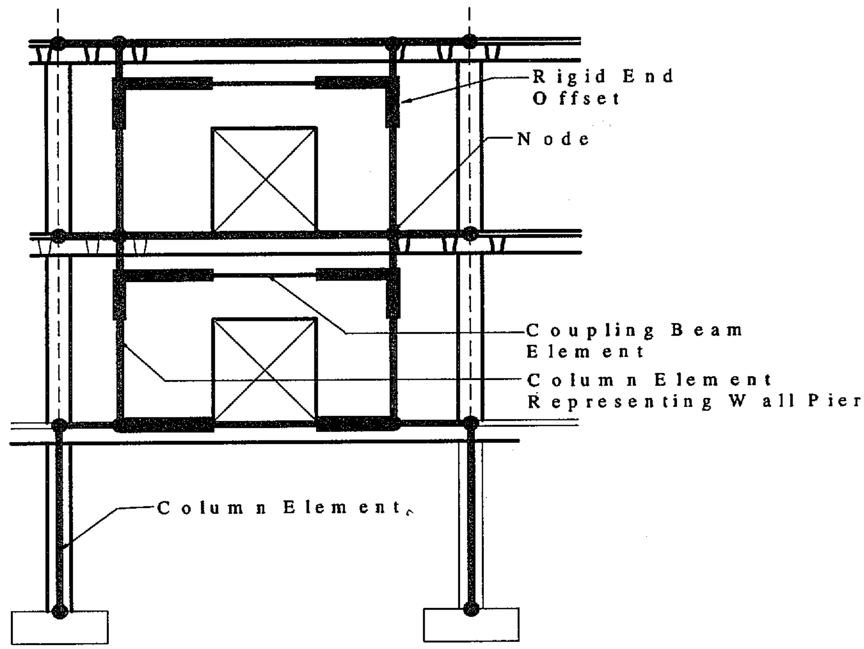


Figure 7-10 Mathematical Model of Coupled Shear Wall

7.4.2.2 Component Force-Displacement Behavior

Component force-displacement relations, FEMA 273 and Sections 4.3 and 4.4 of FEMA 306

Component force-displacement curves are developed for each of the typical wall components using the generalized force-displacement curves from Figure 6-1 of FEMA 273. The acceptance limits for the coupling beam components are based on Table 6-17 of FEMA 273 for the case of "nonconforming", transverse reinforcement, and shear exceeding $6t_w l_w \sqrt{f_c'}$. The pre-event shear-strength-to-chord-rotation relationship is shown in Figure 7-12(a). Also shown in this figure are the points representing the displacement limits for immediate occupancy and life safety performance.

Component modeling for pre-event condition, Section 4.4.3.1 of FEMA 306

The initial slope of the component force/deformation curves is based on the initial elastic stiffness of the component. The pre-event structure is modeled using the effective initial stiffness values recommended in Table 6-4 of FEMA 273. Walls and coupling

COEFFICIENT AND CAPACITY SPECTRUM METHODS

Either of two methods are recommended for establishing displacement demands for a nonlinear static analysis: the coefficient method and the capacity spectrum method. A description of these methods is included in ATC 40. The coefficient method is also described in FEMA 273, and the coefficient and capacity spectrum methods are described in FEMA 274. Although either method may be used, it is essential for a valid comparison that the same method be used to assess the performance of the pre-earthquake, post-earthquake, and repaired structure, as outlined in FEMA 306.

In this example, the coefficient method is used. In this method, a target displacement, d_t is calculated and compared to the displacement of a control node, generally located at the roof. The target displacement is determined by multiplying a set of coefficients times a function of the effective building period and the spectral acceleration.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

To use the coefficient method, the nonlinear static analysis must be conducted in order to construct the pushover curve. The pushover curve can be presented as spectral acceleration versus spectral displacement or as base shear versus roof displacement. Once the pushover curve is constructed, an equivalent bilinear curve is fitted to approximate the actual curve. The equivalent bilinear curve is then used to obtain the effective stiffness of the building and the yield base shear needed for calculating the target displacement.

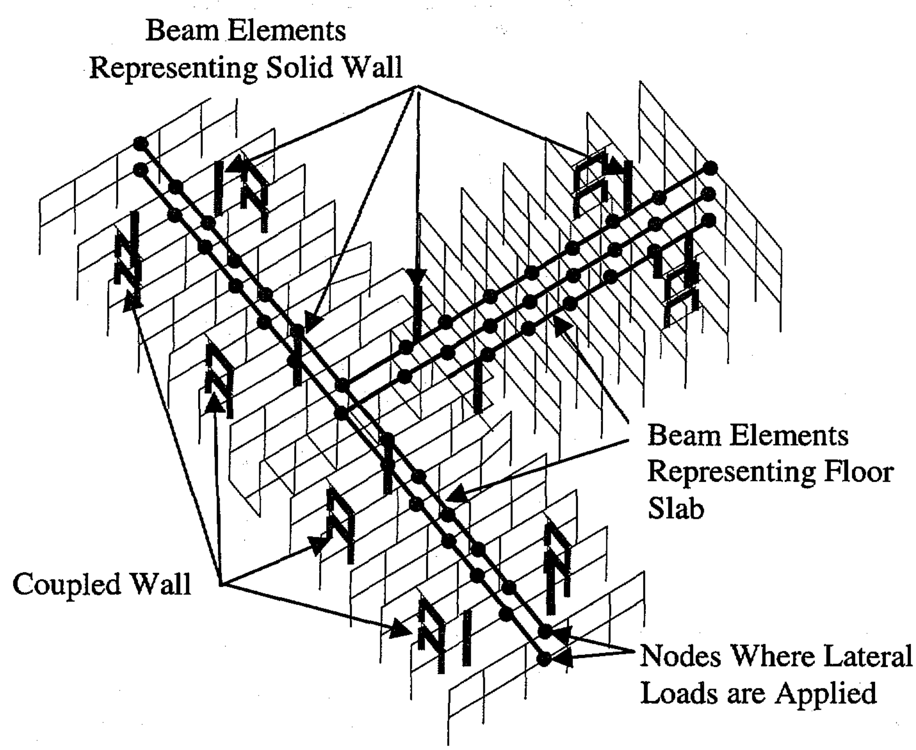


Figure 7-11 Mathematical Model of Full Building

beams are given a flexural rigidity of $0.5E_cI_g$. The basement columns of the structure, which support the discontinuous walls, are given a flexural rigidity of $0.7E_cI_g$. As recommended in FEMA 273, the shear rigidities of all components are set equal to gross section values.

The post-event structure is modeled with stiffness values multiplied by the λ_k factors recommended in FEMA 306. Heavily damaged coupling beams have their stiffness reduced to 20 percent ($\lambda_k = 0.2$) of the pre-event value. Moderately damaged coupling beams have their stiffness reduced to 50 percent ($\lambda_k = 0.5$) of the pre-event value. For the solid shear walls, where damage is classified between Insignificant and None, stiffness is reduced to between 80 percent to 100 percent of the pre-event stiffness depending on the amount of cracking.

The horizontal plateau of the component force/deformation curves is based on the strength of the governing behavior mode. For the pre-event structure, the strength is based on calculations as illustrated in Section 7.2.4 of this example. For the post-event structure, the pre-event strength is multiplied by the λ_Q factors recommended in FEMA 306. Heavily damaged coupling beams have their strength reduced to 30 percent of the pre-event value. Moderately damaged coupling beams have their strength reduced to 80 percent of the pre-event value. For components where damage is classified either Insignificant or None, the strength is not reduced. Figure 7-12(b) shows the force-deformation curves for the moderately and heavily damaged coupling beams.

7.4.2.3 Foundation Rocking

Since the governing behavior mode of the solid concrete walls is identified to be foundation rocking, this behavior is incorporated into the pushover analysis. To model the rocking, the stiffness of the lower story wall elements is reduced when the shear force in those elements reaches the shear force that causes rocking. Once the wall element in the model had started to overturn in the analysis, the stiffness is adjusted so that the wall

NONLINEAR ANALYSIS USING LINEAR ANALYSIS COMPUTER PROGRAMS

Currently, there are few commercially available computer programs for direct implementation of the nonlinear analysis required for a pushover analysis. Many of the nonlinear programs available are sophisticated but can be expensive and difficult to use. For many buildings, a linear elastic analysis program can be used to assess iteratively the nonlinear behavior of the building.

There are two ways to implement a nonlinear static analysis using a linear computer program. Both methods are based on adjusting the stiffness of an element once the analysis indicates that the element has reached its yield level. One method uses the tangential stiffness of the element at the displacement level above yield; the other uses a secant stiffness. The figures below depict the difference between the two methods.

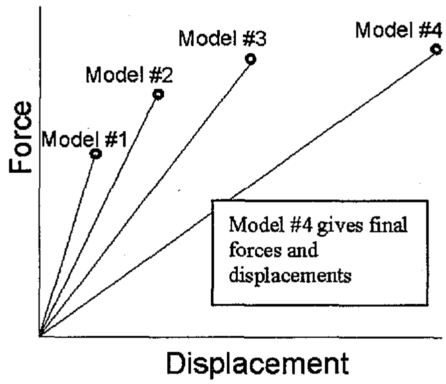


Figure ii – Secant Stiffness Method

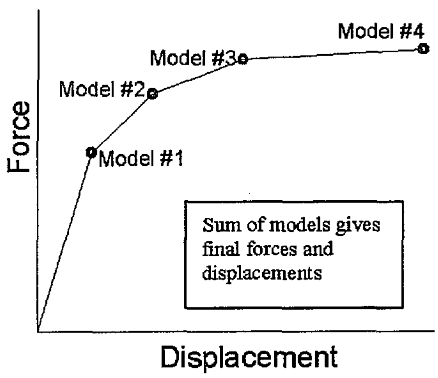


Figure i – Tangential Stiffness Method

The tangential stiffness method is described in detail in ATC 40 (ATC, 1996). Lateral forces are applied to the building and proportionally increased until an element reaches its yield level. A new model is then created in which the yielding component has its stiffness reduced to zero or a small post-yield value. An incremental load is applied to the new model until another component reaches its yield level. The process continues until a complete mechanism has formed or until the maximum displacement level of interest has been reached. The sum of forces and deformations of each of the incremental models then represent the global behavior of the structure.

In the secant stiffness method, lateral forces are applied to the building and proportionally increased until a component reaches its yield level. A new model is then created in which the yielding element has its stiffness reduced by a value chosen to produce the correct post-yield force in the component. The new model is then rerun at the same force level, and components are checked to verify that the force in the component has not exceeded, or reduced significantly below, its yield level. If necessary, the stiffness of the yielding element may need to be adjusted so that the force in that element is approximately equal to the post-yield force level. Other elements need also be checked since they may be resisting additional load no longer resisted by the yielding element. After iterating until all elements are at approximately the correct force level, a new model is created at a larger lateral force level. The process is repeated at each force level. The behavior of the structure and each element at a given force level is represented directly by the behavior of the appropriate model, rather than combining the results of several models.

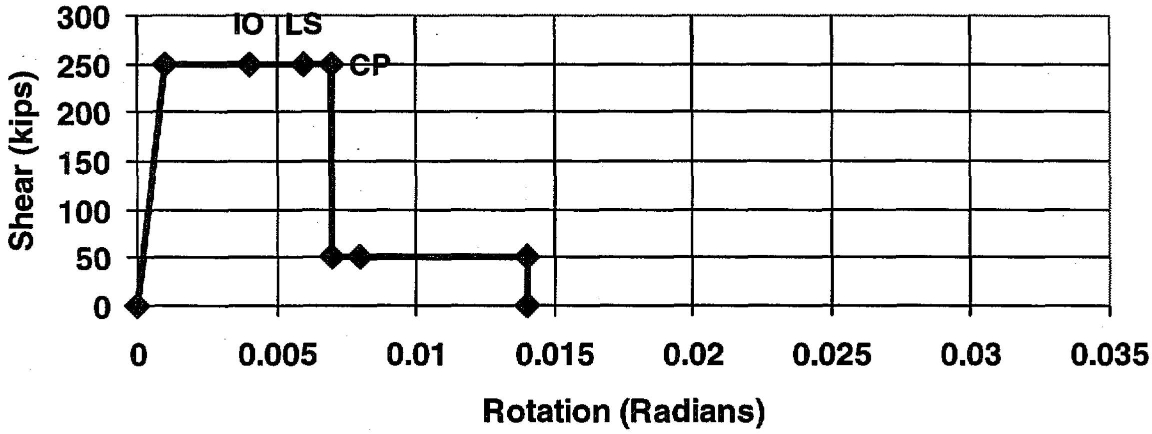
would resist about 10 to 20 percent more shear force than that calculated to cause overturning. This adjustment is made to account for the additional dead weight of the structure that the wall would pick up once it started to uplift. The amount of additional overturning resistance in the wall is based on the shear and moment capacity of the beams framing into the wall.

7.4.3 Force-Displacement Capacity (Pushover Analysis) Results

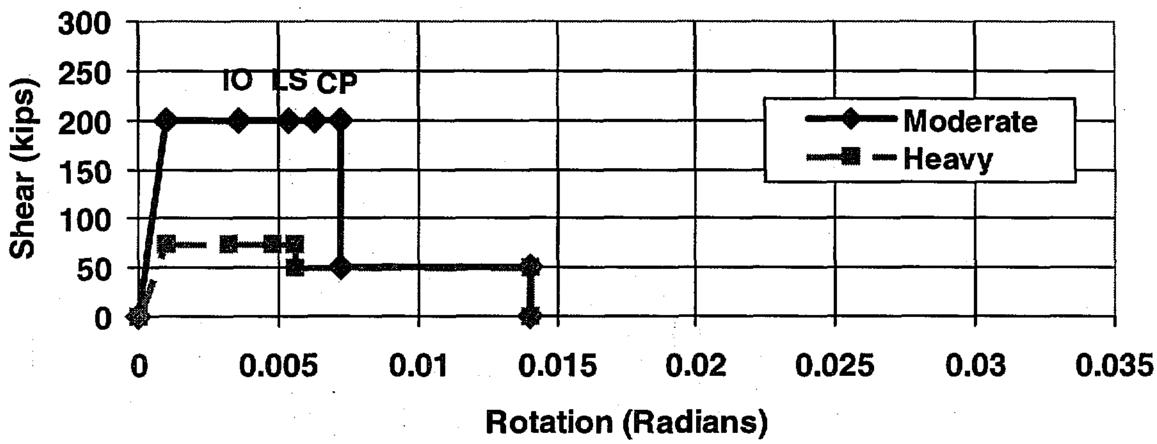
7.4.3.1 Pre-Event Structure

The results of the pushover analysis indicate the progression of displacement events to be as follows for East-West loading (See Figure 7-2 for wall locations):

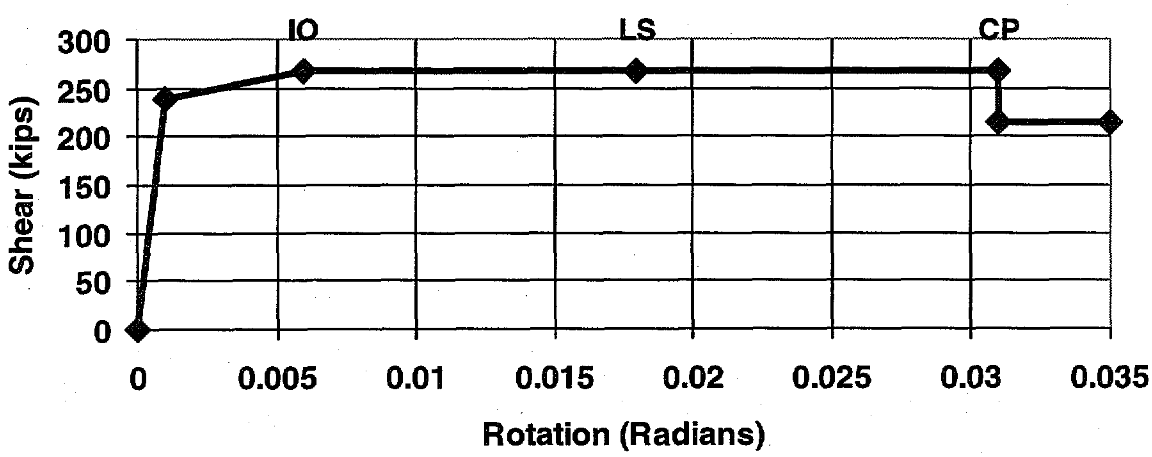
- Initially the two solid walls on lines 7 and 10 between lines C and D reach their rocking capacity.



a) Pre-event



b) Post-event



c) Replacement Coupling Beam with Diagonal Reinforcement

Figure 7-12 ***Component Force-Displacement Curves for Coupling Beams***

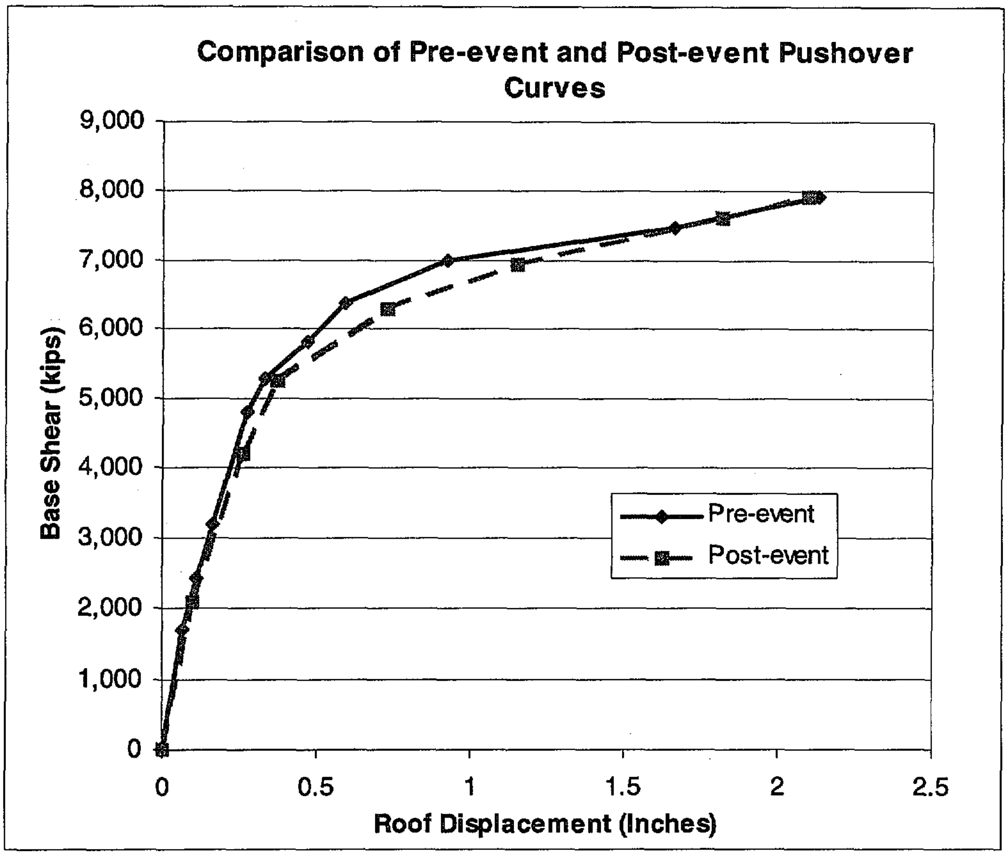


Figure 7-13 ***Comparison of Pre-event and Post-event Pushover Curves***

- When the solid walls between lines C and D are softened, the solid walls on lines 2 and 15 between lines B and C, and between D and E at the first floor pick up additional force and reach their rocking capacity.
- As the solid walls are softened, the coupled walls on lines 7 and 10 between lines L and M resist more force. The first floor coupling beam picks up more force than the second floor coupling beam and reaches its shear capacity first.
- Additional coupling beams reach their capacity and the solid walls continue to rock as the displacement of the structure is increased.
- The approximate target roof displacement is reached after the coupling beams have exceeded their *collapse prevention* acceptability limit, requiring a reduction in their capacity.

As shown in Figure 7-13, the pushover analysis indicates that global nonlinearity begins at a base shear of approximately 5000 k. As lateral displacements increase, the base shear climbs to about 8000 k. Since 10% of the total is applied at the first floor and is transmitted directly into the foundation, the force resisted by the structure above the first floor prior to global nonlinearity is about 4500 k. Allowing for some increase in capacity to reflect rocking behavior more accurately (see Section 7.4.2.3), this agrees well with the hand-calculated capacities of the walls summarized in Tables 7-1 and 7-3. The applied load in excess of the capacity of the walls is resisted by the columns. The magnitude of the increased load is compatible with the capacity of the columns. In the analysis, the first story coupling beams are the first element to reach the immediate occupancy and life safety acceptability limits. The component deformation limit for immediate occupancy occurs when the roof displacement reaches about 0.65 inches and that for life safety is reached at about 0.88 inch. These displacements are taken as the displacement capacity d_c, as defined in FEMA 306.

The progression of damage shown in the analysis is consistent with the observed damage.

7.4.3.2 Post-Event Condition

Modeling of the post-event condition, Section 4.4.3.2 of FEMA 306

For the post-event structure, the progression of displacement events is essentially the same as that outlined for the pre-event structure. The results of the post-event pushover analysis are shown in Figure 7-13. In this analysis, the first story coupling beams reach the immediate occupancy acceptability limit at a roof displacement of 0.47 inches; the beams reach the life safety limit at a roof displacement of 0.66 inches. These values are used for d'_c.

7.4.3.3 Comparison of Force-Displacement Capacity Curves (Pushover Curves)

The performance of the post-event building was slightly different than the pre-event performance; the overall building is softer since more deflection is obtained for the same magnitude of applied load. The reduced stiffness of the damaged components causes the global reduction of stiffness of the post-event structure. The Moderate and Heavy damage to some of the components corresponds to a reduction in their strength. At larger displacements (greater than about 1.5 inches) the response of the pre-event and post-event structures are essentially the same.

7.4.4 Estimation of Displacement, d_e, Caused by Damaging Earthquake

The accuracy of the structural model of the building can be verified by estimating the maximum displacement, d_e, that was caused by the damaging event. This is done in two ways. If the data were available, actual ground motion records could be used to predict displacement analytically. Secondly, the pushover curve in conjunction with component capacity data could be used to estimate displacements from the observed damage.

In this case, a spectrum from recorded ground motion at a site approximately 1.5 mi. from the building was available (see Figure 7-14). *FEMA 273* (equation 3-11) uses the displacement coefficient method to estimate maximum displacement from spectral acceleration as follows:

$$d_e = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{7-1}$$

In this expression the coefficients C_0 to C_3 modify the basic relationship between spectral acceleration and dis-

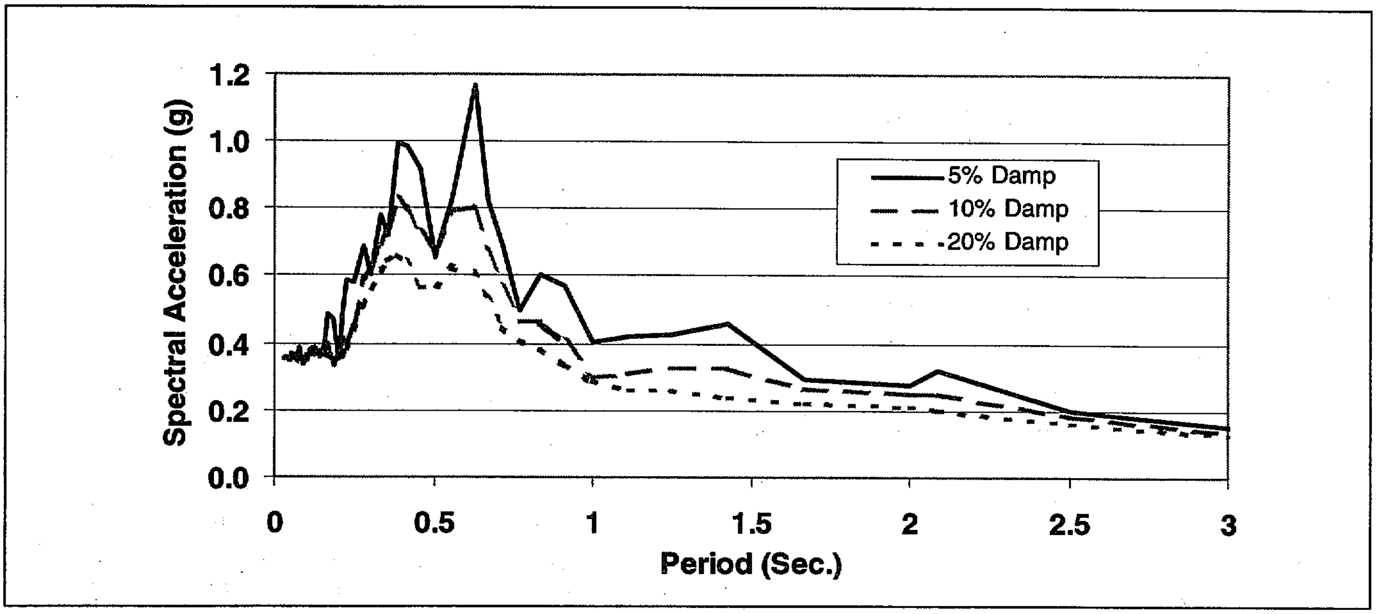


Figure 7-14 ***Response Spectra from Damaging Earthquake***

placement for an elastic system as a function of the effective period of the structure, T_e. The effective period for the pre-event structure is approximately 0.3 sec. The spectral acceleration for this period from Figure 7-14 would be approximately 0.5 to 0.6 g producing an elastic spectral displacement between 0.4 and 0.5 in.

The coefficient C_0 converts spectral displacement to roof displacement and has an approximate value of 1.25 for two- and three-story buildings.

For short-period buildings, the maximum inelastic displacement often is greater than the elastic. *FEMA 273* provides the following expression C_1 to adjust conservatively from elastic to inelastic:

$$C_1 = \frac{\left(1.0 + (R-1.0)\frac{T_0}{T_e}\right)}{R} \quad \text{where } R = \frac{S_a}{\left(\frac{V_y}{W}\right)}\frac{1}{C_0} \qquad (7\text{-}2)$$

In these expressions, V_y/W is the effective base shear at yield as a portion of the building weight, or about 0.28 in this case. This would result in an R-factor of approximately 1.4 to 1.7. The point where the spectral acceleration transitions from the acceleration to velocity controlled zone occurs at a period of around 0.5 to 0.6 sec. These values would combine to result in a coefficient C_1 of around 1.2 to 1.4.

The coefficient C_2 accounts for the shape of the hysteresis curve and is equal to 1.0 in this case. The coefficient C_3 accounts for dynamic P-Δ effects and is also equal to 1.0 for this case.

Combining all of the coefficients and the elastic spectral displacement results in an estimate for the maximum displacement at the roof, d_e, of between 0.6 to 0.9 in.

From the damage observations, one of the first-floor coupling beams in the east-west direction appeared to reach its capacity, since a severe crack had developed and a transverse bar had buckled. Shear cracking had also developed in the wall piers adjacent to the coupling beams.

From the pushover analysis, at displacement demands between 0.3 inches and 0.5 inches, the coupling beams reach their capacity. The pushover analysis also indicates that the first floor coupling beam would be the first to reach its capacity, which is verified by the observations. Since only the first floor beams were heavily damaged, the displacement demand of the damaging event should not have been much greater than 0.5 in.

The difference between the analytical estimate of d_e and the estimate from the model and observed damage is not large. The difference is acceptable because the building is farther away from the epicenter than the site where the motion was recorded, and actual recorded building response is usually less than that which is predicted analytically. Based on the comparison there is no need to adjust the structural model.

7.4.5 Displacement Demand

7.4.5.1 Estimate of Target Displacement

Estimating the target displacement can be an interactive process. The nonlinear static analysis produces a force-displacement pushover curve covering the displacement range of interest. Based on the procedures of FEMA 273, an equivalent bilinear curve is fitted to the pushover curve and a yield point is estimated.

Using this yield point and the associated effective period, the target displacement is calculated using the coefficient method. Given the calculated target displacement, the equivalent bilinear curve can be refitted, adjusting the yield point, and giving a new target displacement. The revised target displacement is close to the original estimate so further iteration is not needed.

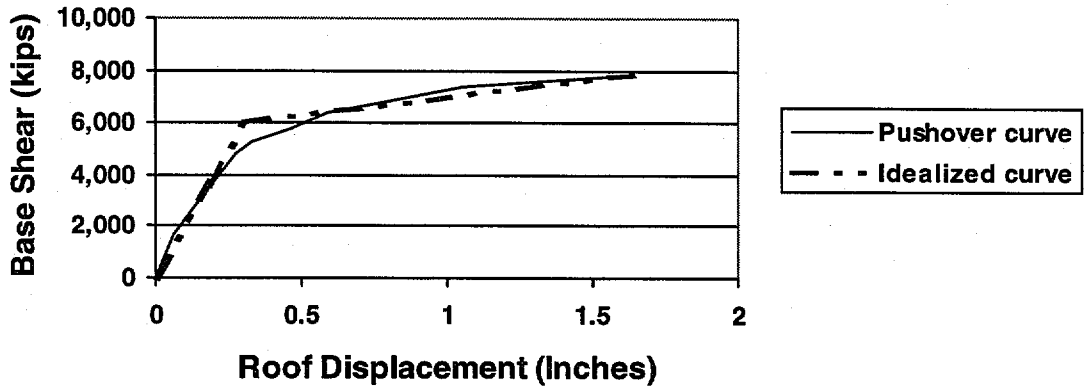
EXAMPLE CALCULATIONS FOR PRE-EVENT AND POST-EVENT DISPLACEMENT DEMANDS

Section 5.4 of FEMA 306 describes the procedures for calculating the displacement demand for both the pre-event and the post-event structures. The pre-event and post-event pushover curves for this example are shown in Figure 7-13. For this example, the coefficient method is used to calculate the target displacements and FEMA 306 procedures are used to determine the corresponding displacement demands.

Pre-Event Target Displacement, d_d

An idealized bi-linear capacity curve for the pre-event structure is developed to approximate the actual pushover curve. Based on this idealized curve, the yield level base shear V_y is 6000 kips and the yield level displacement D_y is 0.31 inches. The effective stiffness K_e then becomes 19,400 kips/inch.

EXAMPLE CALCULATIONS FOR PRE-EVENT AND POST-EVENT DISPLACEMENT DEMANDS (continued)



Comparison of idealized bilinear curve to pushover curve

The initial period T_i is 0.25 seconds taken from the initial structural model. The effective period is calculated to be 0.30 seconds using the ratio of the initial to the effective stiffness.

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} = 0.25\sqrt{\frac{27900}{19400}} = 0.30$$

The spectral acceleration S_a, based on the life safety earthquake response spectra at the effective period is 1.0 g.

The coefficients are:

$C_0 = 1.25$	for a 2-to-3-story building
$C_1 = 1.58$	using the equation for T_e in the constant acceleration region of the spectrum
$C_2 = 1.0$	
$C_3 = 1.0$	

Thus the target displacement from Equation 3-11 of FEMA 273 is:

d_t = 1.25 (1.58) (1.0g) (386 in/sec^{2}g) (0.30)^2/4π^2
= 1.68 inches

This value is assigned as d_d, the maximum displacement in its pre-event condition.

Post-Event Target Displacement, d'_{d1}

There are two values for the post-event displacement demand that need to be calculated. The first value, d'_{d1} uses the pre-event effective stiffness and the post-yield stiffness for the post-event curve to calculate a target displacement. In this example, the slopes of the post-yield curves for the pre-event and post-event conditions are similar. Therefore, the target displacements will be essentially the same. The value for d'_{d1} will be taken as the pre-event demand displacement, which is 1.68 inches.

Post-Event Target Displacement, d'_{d2}

Considering the post-event pushover curve, the effective stiffness K_e, with V_y = 5600 and D_y = 0.32 is 17,500. The initial and effective periods are 0.25 seconds and 0.31 seconds.

The damping coefficient β for the post-event structure is calculated to be 0.06 based on Equation 5-3 of FEMA 306, due to the change in the post-event effective stiffness. The damping adjustments for the response spectrum (B_s and B_1), interpolating from Table 2-15 in FEMA 273, are 1.06 and 1.04 respectively. This changes the spectral acceleration for the post-event structure to 0.97.

The value for C_1 becomes 1.55, and the other coefficients are the same as for the pre-event condition. Using these values, the new target displacement is calculated as:

d_t = 1.71 inches

This value is assigned as d'_{d2}.

The displacement demand from the damaging earthquake d_e was estimated to be 0.6 inches. Since d'_{d1} is greater than d_e, the displacement demand for the post-event structure d'_d is equal to d'_{d1}, which is 1.68 inches.

The target displacement d_d for the pre-event structure for life safety performance against the 475-year-return-period earthquake, based on the coefficient method calculations, is 1.68 inches. The displacement demand for immediate occupancy after the 100-year earthquake is 0.97 inches.

Displacement demand, Section 4.4.4 of FEMA 306

Calculations (see sidebar on previous page) indicate that displacement demand for the post-event structure is essentially the same as for the pre-event structure.

7.4.5.2 Effects of Damage on Performance

The changes in displacement capacity and displacement demand caused by the effects of damage are summarized in Table 7-7. The Performance Indices, P and P', in Table 7-7 are the ratios of the displacement capacity, d_c or d_c', to displacement demand, d_d or d_d', as defined in *FEMA 308*. The displacement capacities calculated in Section 7.4.3 are based on the assumption that the coupling beams are primary components. *FEMA 273* allows coupling beams to be treated as secondary members. Since the global capacity is controlled by the acceptability of the coupling beams, the displacement capacities are determined again assuming that the coupling beams are secondary components and the results are included in Table 7-7. The global displacement capacity, although higher for Life Safety, is still controlled by the coupling beams. The relative change in Performance Index is similar in both cases, indicating that the effects of damage are the same.

The Performance Indices for both the pre-event and post-event structures are less than one for both performance objectives, indicating that the objectives are not met. The effects of damage can be quantified by identifying restoration measures to return the Performance Index to its pre-event value, as outlined in the following sections. The actual course of action to accept, restore, or upgrade the damaged building is a separate consideration for the owner and the local building authority.

7.4.6 Analysis of Restored Structure

7.4.6.1 Proposed Performance Restoration Measures

The primary difference between the pushover models of the pre-event building and the post-event building is the performance of the coupling beams. In their post-earthquake condition, the coupling beams were considered to have less stiffness and strength than in their pre-event condition. The displacement limits were also reduced by the λ_D factor of 0.7. This resulted in the overall reduced stiffness, strength, and displacement capacity of the structure.

To restore the overall performance of the building, various schemes could be investigated, for example, the addition of new concrete walls without repairing damaged components. In this case however, the most straightforward repair appears to be the same component-by-component restoration considered in the direct method. This principally involves the repair of the damaged coupling beams. The coupling beams would be repaired as suggested by the Component Guides in FEMA 306 for the RC3H components. The moderately

Table 7-7 Performance Indices for Pre-event and Post-event Structures

	Displacement Capacity (inches)		**Displacement Demand (Inches)**		**Performance Index (*Capacity/Demand*)**	
	Life Safety	*Immediate Occupancy*	*Life Safety*	*Immediate Occupancy*	*Life Safety*	*Immediate Occupancy*
Coupling beams treated as primary components						
Pre-event	$d_c = 0.88$	$d_c = 0.65$	$d_d = 1.68$	$d_d = 0.97$	$P = 0.52$	$P = 0.67$
Post-event	$d_c' = 0.66$	$d_c' = 0.47$	$d_d' = 1.68$	$d_d' = 0.97$	$P' = 0.39$	$P' = 0.48$
Coupling beams treated as secondary components						
Pre-event	$d_c = 1.00$	$d_c = 0.65$	$d_d = 1.68$	$d_d = 0.97$	$P = 0.60$	$P = 0.67$
Post-event	$d_c' = 0.76$	$d_c' = 0.47$	$d_d' = 1.68$	$d_d' = 0.97$	$P' = 0.45$	$P' = 0.48$

damaged coupling beams are repaired by injecting the cracks with epoxy. The heavily damaged coupling beams are repaired by removing the damaged coupling beams and replacing them with new coupling beams. Each new coupling beam will be designed using the provisions of the current building code, which requires diagonal reinforcing bars be installed as the primary shear resistance. A detail of the potential repair is shown in Figure 7-8.

7.4.6.2 Analysis Results

The moderately damaged coupling beams are "repaired" in the model by revising their stiffness and strength based on the Component Damage Classification Guides. The heavily damaged coupling beams that were replaced are given stiffness values for initial, undamaged elements and displacement capacities as in FEMA 273 for flexure-governed beams with diagonal reinforcement, as shown in Figure 7-12(c). The stiffness of the moderately damaged coupling beams is restored to 80 percent of the pre-event stiffness. The strength and displacement limits are restored to the pre-event values. The strength and stiffness of the other components in the model are unchanged from their post-event condition. The pushover analysis is then conducted using the same procedures and load patterns.

The progression of displacement events for the repaired structure is similar to that for the pre-event structure except that the replaced coupling beam does not reach its collapse prevention displacement limit. Figure 7-15 shows the pushover curve for the repaired structure. Also shown on this curve is the pre-event pushover curve. The overall behavior of the repaired structure closely matches that of the pre-earthquake structure, as it was designed to do. The ratio of displacement capacity to demand, d_c^* / d_d^*, is 0.53 for the life safety performance level and 0.66 for immediate occupancy, which are the same as those for the pre-event performance.

The displacement capacity for the repaired structure is governed by the component deformation limits of the coupling beams that were not replaced. Note that an effective upgrade measure might be to replace all coupling beams, as this would greatly increase global displacement capacity.

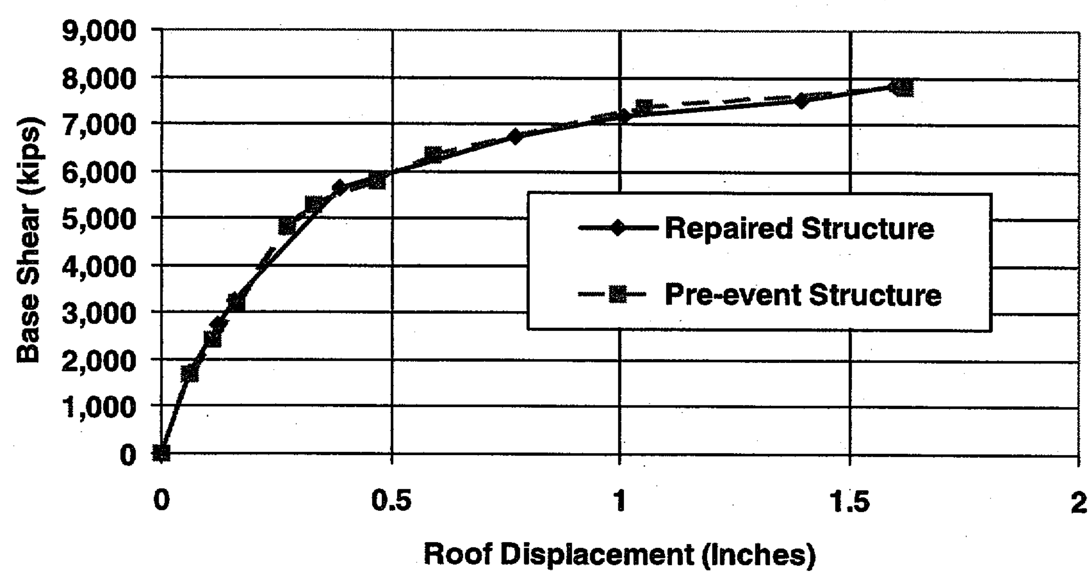


Figure 7-15 Comparison of Pre-event and Repaired Pushover Curves

7.4.7 Performance Restoration Measures

7.4.7.1 Structural Restoration Measures

Hypothetical repairs for relative performance method, Section 4.5 of FEMA 306

Based on the relative performance analysis, replacing the three heavily damaged coupling beams and injecting the cracks in the moderately damaged coupling beams restores the performance of the structure. The volume of reinforced concrete coupling beams to be removed is estimated to be about 41 cubic feet per coupling beam. The length of shear cracks to be injected in the moderately damaged coupling beams is estimated to be 100 feet.

7.4.7.2 Nonstructural Restoration Measures

The Component Guides for the type RC1B components indicate that if cracks are less than 1/8 inch, the damage can be classified as Insignificant, and therefore structural repairs are not necessary. Two of the wall components had cracks that exceeded 1/16 inch. These wall components will have all of the cracks exceeding 1/16 inch repaired by injection with epoxy. The total length of these cracks is estimated to be about 22 feet.

The wall components with visible cracks will be repaired by patching the cracks with plaster and painting the entire wall. This repair is only intended to restore the visual appearance of the wall. Restoration of other nonstructural characteristics, such as water tightness and fire protection, is not necessary.

7.4.7.3 Summary of Restoration Measures and Costs

Table 7-8 summarizes the repairs and estimated costs. Additional costs related to inspection, evaluation, management, and indirect costs may also be involved.

7.5 Discussion of Results

7.5.1 Discussion of Building Performance

The example building contains some typical features found in older concrete wall buildings, such as lightly reinforced concrete elements and discontinuous wall elements. Although the building was designed adequately according to the building code at the time, the design would not be appropriate by current building codes. Because of the improvement in seismic design provisions over the years, it is expected that the building, in its pre-event condition, would not meet the life safety performance level of FEMA 273.

The weak link in the building, as determined by analysis and confirmed with the field observations, is the shear capacity of the coupling beams. Although the analysis indicates that foundation rocking of the solid walls is probably the initial nonlinearity in the building, the rocking of the walls is not detrimental to the global behavior under the anticipated seismic demands.

In the section of the building in which the coupling beams were damaged, the coupled shear walls are discontinuous and are supported by columns at the ends of the walls. Normally, columns supporting discontinuous walls are susceptible to high compressive stresses, and consequently reduced ductility capacity, as the wall overturns. During the pushover analysis, the forces in the columns supporting the coupled walls remained within their capacity. The reason the columns were not overstressed is that the coupling beams acted as fuses for the coupled wall element. The overturning force in the columns could not be greater than the shear capacity of the coupling beams. If the strength of the replaced coupling beams is too large, the overturning force generated could cause failure of the columns below the

Table 7-8 Restoration Cost Estimate by the Relative Performance Method

Item	Unit Cost (1997 Dollars)	Quantity	Cost (1997 Dollars)
Epoxy Injection	$25.00 /lin ft	122 ft	$3,050.
Coupling Beam Removal and Replacement	$74.00 / cu ft	122 ft^3	$9,028.
Patch and paint walls	$0.60 /sq ft	10,175 ft^2	$6,105.
Replace ceiling tiles	$2.00 /sq ft	15,000 ft^2	$ 30,000.
General Conditions, Fees, Overhead & Profit (@ 30%)			$ 14,455.
Total			$ 62,638.

wall, resulting in a partial collapse of the building. For this reason, the capacity of the repaired coupling beam was designed to be similar to that of the previous coupling beam.

One of the advantages of the relative performance analysis is the ability to assess the behavior of structure and the influence of the behavior of the individual components on the overall behavior. Strengthening a single component may not produce a significant improvement in the overall performance if the progression of failure shifts to a less desirable mode. The pushover analysis of the repaired structure needs to consider the change in overall behavior caused by the repairs.

Because of the improved performance of the first story coupling beams that were replaced, these beams no longer control the global displacement limit of the structure. The force/displacement capacity of the second story coupling beams in their repaired condition is the same as in the pre-event condition. The displacement demand at which the second story coupling beams reach their acceptability limit is very close to the limit at which the first story coupling beams in the pre-event condition reached their limit. Therefore, the overall performance of the building is not improved substantially. The information gained from these analyses can be used to assess whether an upgrade of the building to improve its performance may be cost effective.

7.5.2 Discussion of Methodology and Repair Costs

This example has illustrated some of the important aspects in the FEMA 306 approach to assessing the earthquake damage to concrete and masonry wall buildings. The example building represents an actual building that experienced a damaging earthquake.

FEMA 306 presents two methods for calculating the loss associated with earthquake damage, the direct method and the relative performance method. These methods are used to determine the loss, which is measured as the cost associated with returning the building to its pre-event performance. In this example, the cost of restoring the performance using the two methods produce the same result, principally because the repairs chosen in the relative performance method match those suggested by the direct method. In other buildings, there can be differences between the results obtained by the two methods.

The Nonlinear Static Procedure described in FEMA 273 is used in the relative performance method to assess the performance of the building in the pre-event, post-event and repaired conditions. This analysis method is relatively new and is still subject to further refinements. This procedure can be time-consuming to implement properly. As the method and the analytical tools become further developed, this method should be easier to implement.

7.6 References

ACI Committee 201, 1994, "Guide for Making a Condition Survey of Concrete in Service," *Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

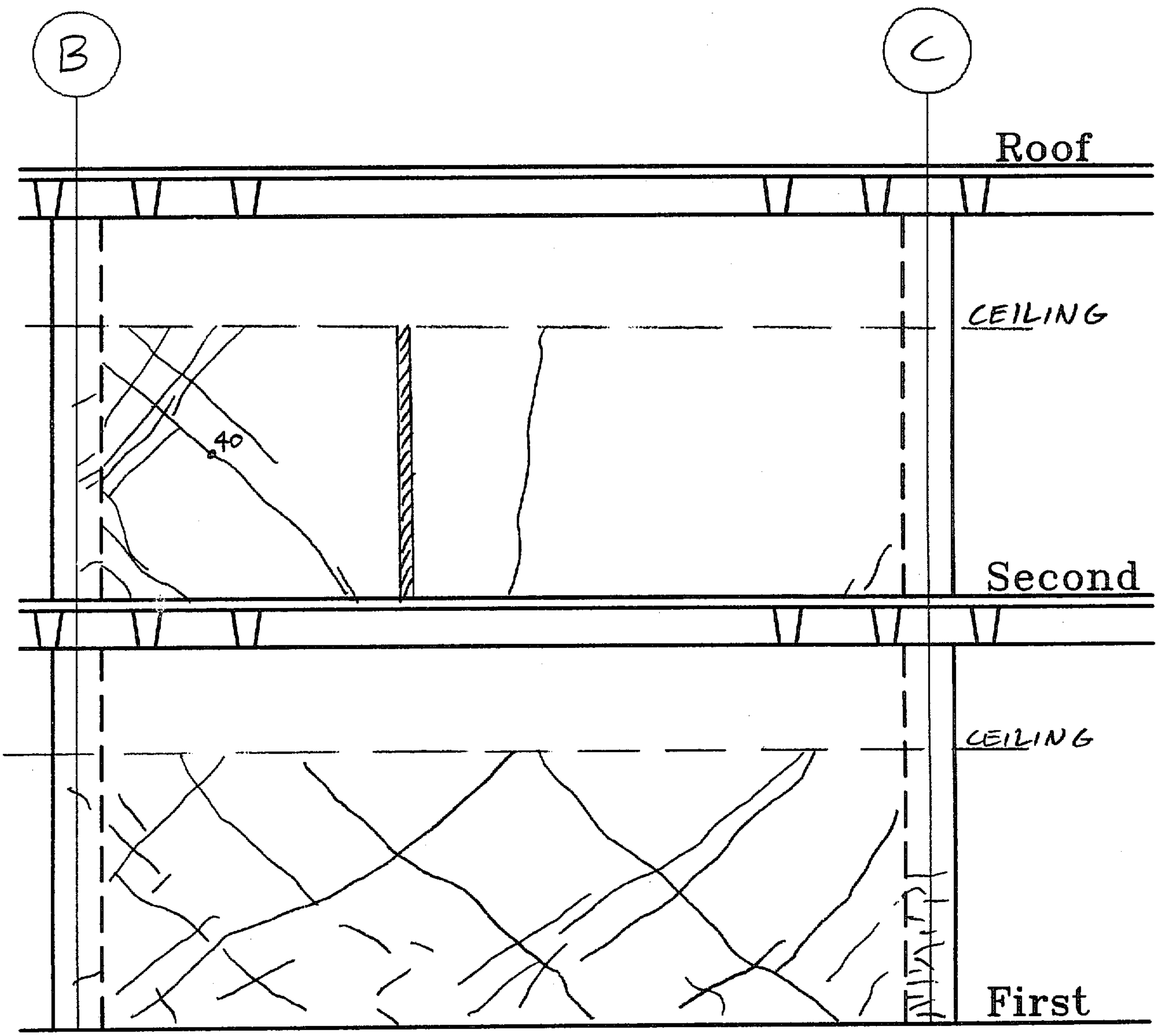
CSI, 1997, *SAP2000: Integrated Structural Design & Analysis Software*, Computers and Structures Inc, Berkeley, California

ATC, 1996, *The Seismic Evaluation and Retrofit of Concrete Buildings,* Applied Technology Council, ATC-40 Report, Redwood City, California.

Appendix A. Component Damage Records for Building Evaluated in Example Application

Component Damage Record D1		
Building Name: Concrete Shear Wall Building	**Project ID:** ATC 43 Example	**Prepared by:** ATC
Location Within Building: **Floor:** $1^{st}/2^{nd}$ **Column Line:** 2 **Component Type:**		**Date:** 24-Sep-97

Sketch and Description of Damage:



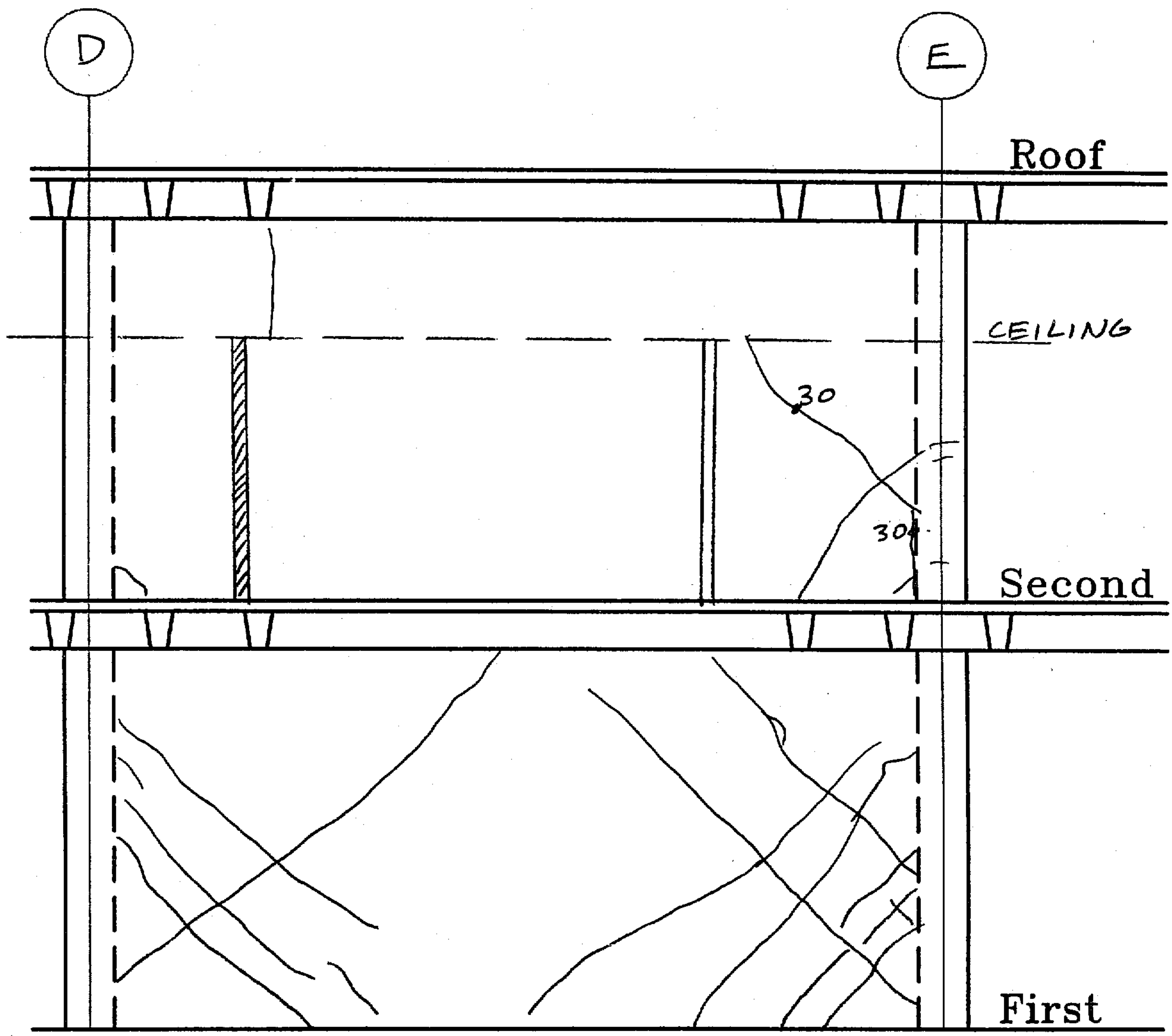
Legend:

	Crack		Spall
30	Crack Width in Mils (0.001 Inch)	NA	Not Accessible
	Crack Previously Filled with Epoxy		Partition
	Crack at Pre-existing Surface Patch		

Component Damage Record D2

Building Name: Concrete Shear Wall Building	**Project ID:** ATC 43 Example	**Prepared by:** ATC
Location Within Building: **Floor:** 1st/2nd **Column Line:** 2 **Component Type:**		**Date:** 24-Sep-97

Sketch and Description of Damage:



Legend:

Crack
Crack Width in Mils (0.001 Inch)
Crack Previously Filled with Epoxy
Crack at Pre-existing Surface Patch

Spall
Not Accessible
Partition